EFFECTS OF LONG TERM MOISTURE STORAGE ON CONCRETE TEST SAMPLES

By:
Jeffrey Stein
SRA International, Inc.
Linwood, NJ
USA
Phone: (609) 485-8180; Fax: (609) 485-4845
Jeffrey.S-CTR.Stein@faa.gov

David R. Brill and Charles Ishee
FAA Airport Technology R&D Branch, ANG-E262
William J. Hughes Technical Center
Atlantic City International Airport, NJ 08405
USA
David.Brill@faa.gov
Charles.Ishee@faa.gov

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ABSTRACT

Concrete flexural strength is the primary thickness design input for rigid airport pavement. Therefore it is necessary to have an accurate estimate of pavement strength not only at the time of construction but also at the time of pavement testing. Researchers have three alternatives for estimating concrete strength at the time of testing, test cast samples which have been cured in the laboratory, test cast samples cured in the field, or cut samples from the pavement immediately after traffic testing.

At the National Airport Pavement Test Facility (NAPTF) during Construction Cycle 6 (CC6), all three test sampling methods were used with varying results for concrete strength depending on the curing method. Three different concrete mix designs were used on the project. All mixes were straight cement, with no supplementary cementitious materials (SCMs). MRS1 was a “low” strength mix with a design flexural strength of 500 psi. MRS2 was a “medium” strength mix with a design flexural strength of 750 psi. MRS3 was a “high” strength mix with a design flexural strength of 1000 psi. The MRS1 concrete mix was made with gravel and sand aggregates. The MRS2 and MRS3 concrete mixes were made with dolomite and sand aggregates. The main difference between MRS2 and MRS3 mixes was the amount of cement.

Laboratory tests were performed around the time of pavement testing which was approximately two years after construction. Samples from MRS2 and MRS3 stored in the moisture curing room for two years had a lower average flexural strength than tests performed at 28 days. The flexural strengths of field samples from MRS1, MRS2 and MRS3 mixes were higher than at 28 days. Sawed beams from MRS1, MRS2 and MRS3 had flexural strength approximately the same as those beams tested at 28 days. Petrographic analysis indicated that MRS2 and MRS3 concrete samples left in the curing room had ASR damage and secondary ettringite formation in ASR induced cracks. The conclusion is that prolonged storage of concrete samples in curing rooms is not recommended since tests made after this time do not reflect the condition of the pavement which has cured in a much drier environment. Field cured samples or saw cut sample are more likely to give a good estimation of concrete strength at the time of testing.

INTRODUCTION

The Airport Technology Research and Development Branch (ATRD) of the Federal Aviation Administration conducted an experiment to determine if high flexural strength Portland Cement concrete had a reduced fatigue life compared to concrete mixes designed according to Advisory Circular 150/5320-6E.[1] The Advisory Circular 150/5320-6E recommends a design flexural strength for P-501 materials to be between 600 to 700 psi at 28 days. Construction Cycle Six (CC6) in the National Airport Pavement Test Facility (NAPTF) was built in three rigid pavement sections: MRS1 had a low target flexural strength Portland Cement Concrete (PCC) mix (FLEX500) of 500 psi, MRS2 had a medium target flexural strength PCC mixture (FLEX750) of 750 psi, and MRS3 had a high target flexural strength PCC mixture (FLEX1000) of 1000 psi. These test sections were built in April and May of 2010, and tested with the NAPTF test vehicle between July 2011 and April of 2012. When the PCC mixes were being placed, the FAA made 391 beam samples and 343 cylinder samples to verify the three mixes met construction requirements, and to be used for a laboratory study on concrete fatigue. The laboratory samples were transported to a commercial testing laboratory and stored in a concrete curing room until they were needed for testing.

Flexural strength data reported by the commercial testing laboratory indicated that all three PCC mixes met or exceeded their design requirements (see Table 1). The remaining beam samples were kept at the commercial laboratory in a concrete curing room until Construction Cycle 6 was ready for traffic loading. During the month of December 2011, the beams were transported to the curing room of the NextGen Pavement Materials Laboratory during the traffic loading of the CC6 rigid pavement sections. In February 2012, beam samples from each of the three mixes were tested for flexural strength to determine the Modulus of Rupture (R), which is required for calculating the stress ratio for concrete fatigue testing. The results of these flexural strength tests nearly two years after the concrete was originally placed is shown in Table 2. The average flexural strength (R) of beam samples of MRS1 was 671 (psi). The average flexural strength (R) of beam samples of MRS2 was 577 (psi) (76% of the 28 day strength). The average flexural strength (R) of beam samples of MRS3 was 645 (psi) (64% of the 28 day strength). The flexural strength of the concrete beam samples from MRS2 and MRS3 had apparently lost strength after nearly two years of moisture curing, whereas the flexural strength of the beam samples from MRS1 had remained the same as the 28 day samples.

The FAA decided to conduct an investigation to determine the in-situ flexural strength of the concrete in the pavement sections of Construction Cycle 6. After the rigid pavement sections were tested to failure, a contractor was hired to saw beam samples from slabs outside the traffic tested areas. The beam samples were immediately taken to the NextGen Pavement Materials Laboratory which is adjacent to the NAPTF and tested according to ASTM C78.

Table 1. Construction Quality Control Data.

CC6 Section	MRS1	MRS2	MRS3
PCC Mix	FLEX500	FLEX750	FLEX1000
7 Day Avg. (psi)	600	729	958
7 Day St. Dev. (psi)	50	71	86
7 Day COV	0.08	0.10	0.09
14 Day Avg. (psi)	602	803	989
14 Day St. Dev. (psi)	59	84	41
14 Day COV	0.10	0.10	0.04
28 Day Avg. (psi)	662	763	1007
28 Day St. Dev. (psi)	48	113	150
28 Day COV	0.07	0.15	0.15

Table 2. Prolonged Moisture Room Storage Flexural Strength.

CC6 Section	MRS1	MRS2	MRS3
PCC Mix	FLEX500	FLEX750	FLEX1000
Age (days cured)	669	660	639
Average Strength, *R* (psi)	671	577	645
Standard Deviation (psi)	47	50	77
COV	0.07	0.09	0.12

STRENGTH INVESTIGATION

A contractor was hired to cut beam samples from the CC6 test pavement after all traffic on the test sections was complete. The contractor cut three six-inch diameter cores from each slab and then used a diamond blade track saw to make two parallel cuts between the core holes. Anchor bolts were fitted to the two concrete samples from each slab. The pavement slice was removed using a chain-hoist and load frame. The pavement slice was rotated onto wooden supports. The track saw was repositioned to cut the pavement slice into 6 × 6 inch beams taken from the middle of the concrete slice. Figures 1, 2 and 3 show concrete beams being cut and removed from CC6 pavement slabs.

Figure 1. Track Saw Cutting Slab

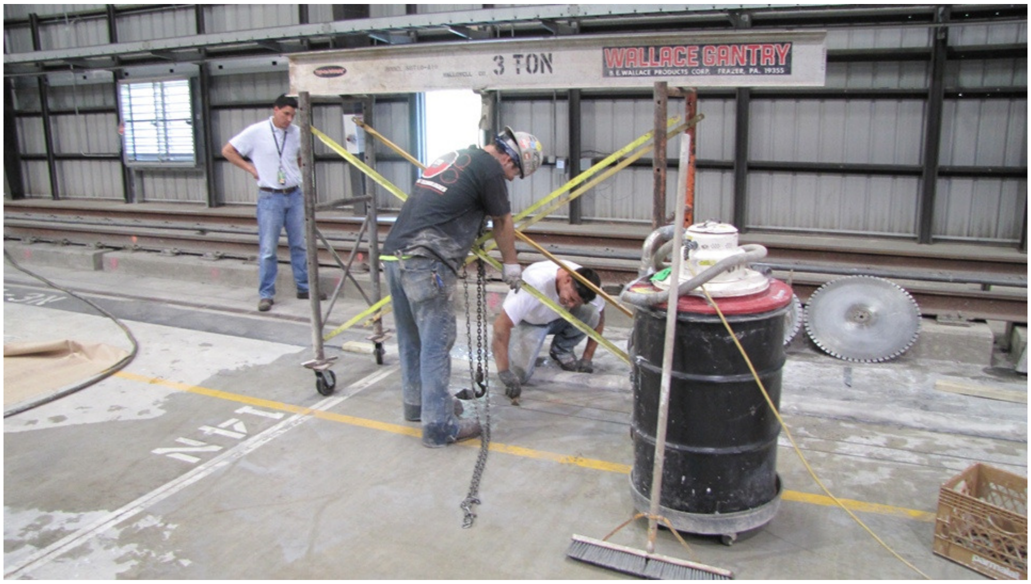

Figure 2. Preparing for Slice Removal

Figure 3. Hoisting the Concrete

Care was taken to maintain the moisture condition of the concrete by wrapping the beams in plastic soon after they were cut. The beams were transported to the FAA NextGen Laboratory and placed in a lime-saturated water tank. All flexural strength tests were run on cut beams after 40 hours of soaking as per ASTM C42. The results of the testing on saw cut beams are shown in Table 3. The average flexural strength (R) of saw cut beam samples of MRS1 was 660 psi, the same as the 28 day strength. The average flexural strength (R) of beam samples of MRS2 was

749 psi (98% of the 28 day strength). The average flexural strength (R) of beam samples of MRS3 was 932 psi (92% of the 28 day strength). The conclusion was that these test results were representative of the flexural strength of the concrete at the time of testing. These values were used for laboratory fatigue testing to determine stress ratios for life cycle analysis of the concrete in the three sections.

Table 3. Flexural Strength of Sawed Beams.

CC6 Section	MRS1	MRS2	MRS3
PCC Mix	FLEX500	FLEX750	FLEX1000
Age (days cured)	775	764	748
Average Strength, R (psi)	660	749	932
St. Dev (psi)	51	30	59
COV	0.08	0.04	0.12

FATIGUE STUDY ON SAW CUT CONCRETE BEAMS FROM CC6

The decision was made to use saw cut beams for the laboratory fatigue study instead of the original cast beams kept in long term moisture storage. The cutting contractor returned to the NAPTF and cut 50 additional beam samples for fatigue testing, 16 beams from MRS1, 18 beams from MRS2, and 16 beams from MRS3. The beam fatigue tests were run in third-point loading configuration at a rate of two cycles per second using a haversine wave form with amplitudes of 90%, 80% and 70% R. The results of the beam fatigue tests performed on saw cut beams are shown in Table 4. Fatigue tests of the laboratory cured cast beams from MRS1 were also performed and included in the data since these beams had shown no strength loss at the time they were tested.

Table 3. Summary of Concrete Beam Fatigue Data.

Section	Stress Ratio	Max Cycles	Min Cycles	Average Cycles	COV
MRS1	0.7	33763	8005	19851	0.52
MRS1	0.8	7534	259	2529	1.15
MRS1	0.9	1013	35	378	0.90
MRS2	0.7	65274	458	17259	1.55
MRS2	0.8	1021	78	321	1.25
MRS2	0.9	582	41	219	1.03
MRS3	0.7	34696	13297	20027	0.50
MRS3	0.8	37458	81	7336	2.03
MRS3	0.9	1047	2	226	1.79
MRS1 Lab Cured	0.7	80373	2277	22066	1.37
MRS1 Lab Cured	0.8	11407	303	2619	1.51
MRS1 Lab Cured	0.9	1242	33	289	1.28

The results of the concrete beam fatigue tests showed no evidence for the theory that higher flexural strength concrete has less fatigue resistance than standard flexural strength (600 to 700 psi) concrete. A plot of the test data for the three concrete mixes is shown in Figure 4. The number of loading cycles to failure of the medium and high strength mixes as a function of flexural strength ratio was similar to the behavior of standard flexural strength concrete mixes, and therefore can use the standard life cycle prediction of Portland Cement Concrete pavements. (See Hilsdorf and Kesler[3], and Fordyce and Yrjanson[4] for details of previous fatigue studies.)

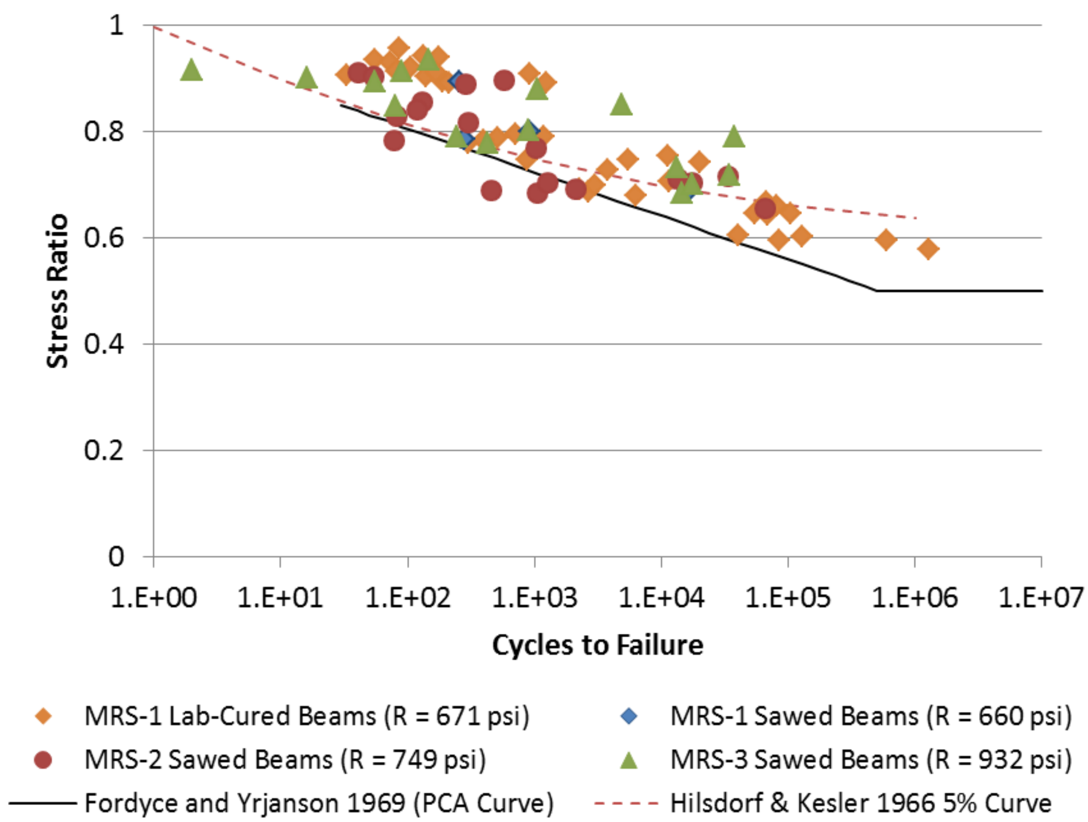

Figure 4. Concrete Beam Fatigue Data for CC6 Test Sections.

FORENSIC STUDY ON EFFECT OF MOISTURE CURING

Although the concrete beam fatigue study was completed using sawed cut beams, the FAA wanted to know why the MRS 2 and MRS3 concrete beams cast during construction and left in the curing room for two years had a reduced flexural strength. Eight additional beams were saw cut from each of the CC6 test sections. Four sawed beams each from MRS1, MRS2 and MRS3 were placed in the laboratory moist curing room and four beams each were placed in a lime saturated water tank. After six months these beams were removed and tested for flexural strength. In addition there were some cast concrete beams that had been left in the facility. These field beams had no exposure to moisture for over two years.

A comparison of the flexural strength of concrete beams subjected to various curing conditions is shown in Table 5. There is a trend in the results that as exposure to moisture increased, the flexural strength of the concrete beams decreased. There was no benefit to soaking the concrete beams in lime saturated water (in fact this reduced the strength slightly). The trend of flexural strength as it is exposed to increasing moisture is shown in Figure 5.

A clue to what may have been occurring to the concrete beams was observed during the flexural strength testing of the beams that had soaked in the lime saturated water tank. The laboratory technician doing the testing noticed that some of the beam appeared to have a white substance growing on the surface (See Figure 6 and Figure 7). The white substance did not appear to be lime, but a white translucent gel which could not be rubbed easily from the surface. The gel was not present on the MRS1 beams, or on the concrete beams left in the curing room. The possibility was that that high alkalinity of the lime saturated water in the tank had promoted the growth of the gel on the sides of the beams. A decision was made to send concrete samples from CC6 to a petrographer for analysis.

Table 5. Comparison of Flexural Strength vs. Moisture.

	Average Flexural Strength, psi			Water Exposure
Curing Condition	MRS1	MRS2	MRS3	(Least to Most)
Field Curing of Construction Beams in NAPT Facility (940 days)	796	912	998	0, Dry
Sawed Beams from Slabs (after 760 days)	660	749	932	1, In-Situ
Lab Curing of Construction Beams (640 days)	671	577	645	2, Wet
Sawed Beams in Curing Room for 6 months (1100 days total)	619	538	638	3, Wet
Sawed Beams Tank Soaking in Lime Saturated Water for 6 months (1100 days total)	564	499	615	4, Wet + High pH

Figure 5. CC6 Beam Flexural Strength vs. Curing Condition.

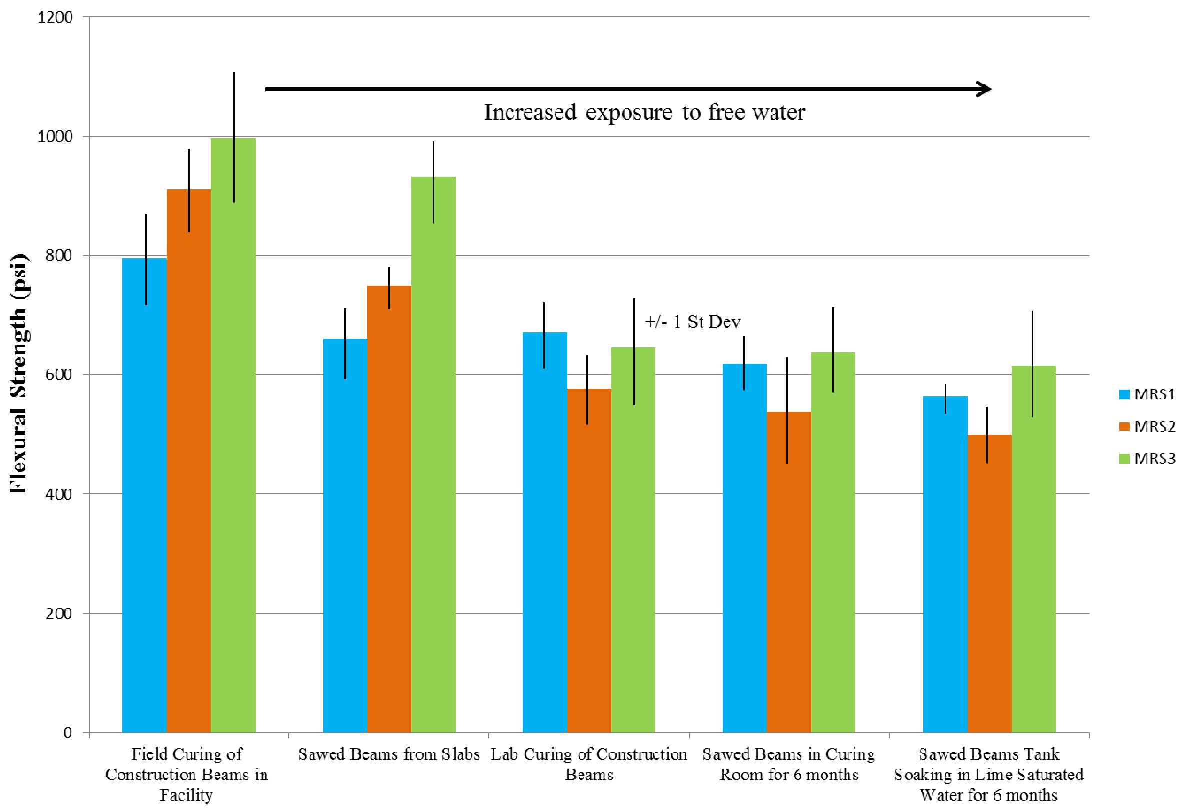

Figure 5. CC6 Beam Flexural Strength vs. Curing Condition

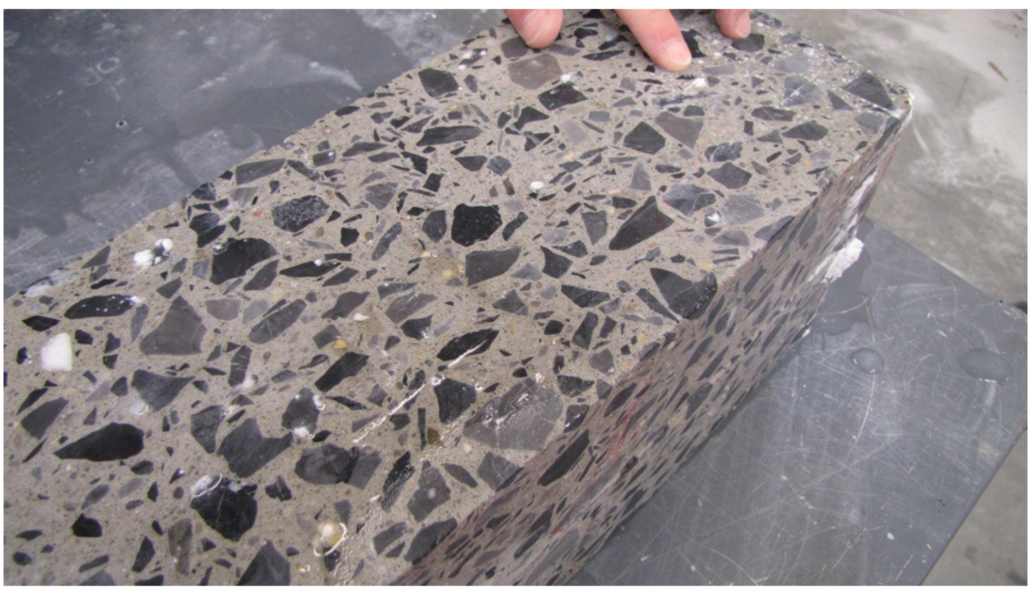

Figure 6. Technician Indicating White Gel Growth.

Figure 7. White Gel Growth on Beam Cut from Slab 39S (MRS3).

PETROGRAPHIC ANALYSIS

Samples of concrete from each CC6 test section, MRS1, MRS2, and MRS3, representing various moisture exposure conditions (field samples, saw cut samples left in the curing room, saw cut samples soaked in lime saturated water, and long-term lab cured cast samples) were sent to a contractor for petrographic analysis. Information about the original mix designs is shown in Table 6. The principal difference in the mixes was the different sources of the aggregates. MRS1 was made using a gravel coarse aggregate (Harmony) and natural sand fine aggregate (Harmony). MRS2 and MRS3 were made using a crushed cherty dark gray dolomite with quartzite particles for the coarse aggregate (Berks) and a natural sand fine aggregate (TS&G). All aggregates used in these mixes had previously been screened for use in concrete pavements by the New Jersey Department of Transportation and were presumed to be minimally alkali reactive.

The petrographic work was performed at the Texas Transportation Institute (TTI) of Texas A&M University. TTI staff cut thin sections from the concrete samples and analyzed the sections using a microscope. The petrographic report by Mukhopadhyay [4] noted the presence of alkali-silica gel from Alkali-Silica Reaction (ASR) in some of the concrete samples, as well as delayed ettringite formation (DEF) in micro-cracks that had likely been caused by the ASR. Alkali-Silica Reaction is a reaction between alkali in the pore water solution of Portland Cement Concrete and silica in the aggregates that causes the formation of a water absorbing, expansive material (gel) that leads to concrete cracking. Ettringite, microscopic crystals composed of calcium, sulfur and aluminum, is a common byproduct of the hydration of Portland Cement. The formation of ettringite is not usually a problem during early stages of hydration before concrete has completely hardened. Ettringite that forms in hardened concrete (delayed ettringite) however, can cause cracking as the crystals expand. A summary of observations taken from the TTI report is shown in Table 7. What is significant from the report is that the concrete samples from MRS2 and MRS3 had more evidence of ASR and DEF, than MRS1. Furthermore the MRS2 and MRS3 samples that were exposed to moisture had evidence of ASR and DEF, whereas the field samples that were in a dry environment in the NAPTF showed no evidence of ASR and DEF.

Table 6. Mix Designs for CC6.

Concrete Mix	MRS1 (FLEX500)	MRS2 (FLEX750)	MRS3 (FLEX1000)
Harmony Gravel No. 57 Coarse Aggregate, Round, lbs/cu.yd.	1550	Not applicable	Not applicable
Berks Stone No. 57 Coarse Aggregate, lbs/cu.yd.	Not applicable	1475	1535
Berks Stone No. 8 Intermediate Coarse Aggregate, lbs/cu.yd.	Not applicable	490	535
Harmony Concrete Sand, lbs/cu.yd.	1414	Not applicable	Not applicable
TS&G Concrete Sand, lbs/cu.yd.	Not applicable	1225	1070
Water, lbs/cu.yd.	325	230	236
Type 1 Portland Cement, lbs	460	500	680
Air, %	6.5	7	4.5
Slump, in.	6	5.5	3.5
SIKA air entrainment agent, oz.	4.5	5	4.5
W/C Ratio	0.71	0.46	0.35

Table 7. Petrographic Analysis of CC6 Samples [2].

Curing Condition	Petrographic Features: MRS1	MRS2	MRS3	Water Exposure (Least to Most)
Field Curing of Construction Beams in NAPT Facility (940 days)	Gravel Aggregates/ potentially ASR reactive, micro-cracking but no ASR gel observed, or delayed ettringite observed	Limestone with chert CA, Siliceous sand FA, No ASR gel, No delayed ettringite (DEF) observed	Limestone with chert CA, Siliceous sand FA, No ASR gel, No delayed ettringite (DEF) observed	0, Dry
Sawed Beams from Slabs (after 760 days)	Not Tested	Not Tested	Not Tested	1, In-Situ
Lab Curing of Construction Beams (640 days)	Gravel Aggregates/ potentially ASR reactive, micro-cracking but no ASR gel observed, ettringite in voids	Limestone with chert CA, Siliceous sand FA, ASR gel, voids and cracks filled with ettringite	Limestone with chert CA, Siliceous sand FA, No ASR gel, but voids and cracks filled with ettringite	2, Wet
Sawed Beams in Curing Room for 6 months (1100 days total)	Gravel Aggregates/ potentially ASR reactive, micro-cracking but no ASR gel observed, ettringite in voids	Limestone with chert CA, Siliceous sand FA, ASR gel, voids and cracks filled with ettringite	No sample available	3, Wet
Sawed Beams Tank Soaking in Lime Saturated Water for 6 months (1100 days total)	Gravel Aggregates/ potentially ASR reactive, micro-cracking but no ASR gel observed, ettringite in voids	Limestone with chert CA, Siliceous sand FA, ASR gel, voids and cracks filled with ettringite	Limestone with chert CA, Siliceous sand FA, ASR gel, voids and cracks filled with ettringite	4, Wet + High pH

CONCLUSIONS

The concrete samples made for Construction Cycle 6 at the NAPTF and stored in a moist curing room were no longer representative of the concrete in the pavement at the time of traffic testing. The concrete in the Test Facility was protected by an enclosure that kept out precipitation, but was not temperature controlled. The concrete beam samples cast at the time of construction were placed in a moist curing room with free water on the surfaces at all times, and kept at a constant temperature between 70 ^{o}F and 77 ^{o}F (ASTM C 31 Standard). The combination of constant moisture and warm temperature facilitates the growth of alkali-silica gel and ettringite crystals. The concrete pavement in the Test Facility was watered on a weekly basis to minimize curling, but the bottom of the slabs was not exposed to water.

The CC6 test sections were constructed from April 26, 2010 to May 18, 2010. Traffic testing of CC6 began July 8, 2011 and was completed April 25, 2012, approximately two years after construction. Flexural strength testing of the laboratory cured beams began February 16, 2012 and was completed February 28, 2012, just a couple months prior to completion of the traffic testing. The sawed beam flexural strength tests were performed from June 4, 2012 and were completed June 11, 2012. The difference in the ASTM C78 concrete beam flexural strength testing results of the laboratory cured beams and the saw cut beams was 172 psi for MRS2, and 287 psi for MRS3, with the saw cut beams having a higher flexural strength in both sections. There was little difference in flexural strength, 11 psi, between the laboratory cured beams and the saw cut beams from MRS1. (See Table 5) However it is clear from the ASTM C78 test results for MRS2 and MRS3, that it cannot be assumed that laboratory cured concrete specimens have the same properties as the concrete in the pavement.

In March 2014, ASTM C78 flexural strength tests were performed on concrete beams left in the curing room that had been cast during construction. The results of these tests are shown in Table 8. Four beams were tested from MRS1. Ten beams were tested from MRS2. Twelve beams were tested from MRS3. The average flexural strength of MRS2 beams was 569 psi, a loss of 194 psi (75% of 28 day strength). The average flexural strength of MRS3 beams was 437 psi (57% of 28 day strength). Even the MRS1 concrete beams which had seen little strength loss after two years in the moisture curing room lost 93 psi (86% of 28 day strength).

Table 8. Flexural Strength of Lab-Cured Beams at 4 Years.

CC6 Section	MRS1	MRS2	MRS3
PCC Mix	FLEX500	FLEX750	FLEX1000
Age (days cured)	1434	1430	1421
Average Strength, R (psi)	596	569	570
St. Dev (psi)	29	33	33
COV	0.05	0.06	0.06

Based upon the research with concrete beams for CC6, it is not recommended that laboratory cured beams be used to determine the strength of a concrete pavement at a particular time. Laboratory cured concrete beams that were cast at time of placement can be used to verify that a concrete mix meets design requirements, since this is a direct comparison to the results of previous tests made with the laboratory cured concrete beams made as part of the mix design procedure. Field cured beams that have been cured in the same conditions for the same time period as the concrete pavement are more likely to represent the actual flexural strength of the concrete in the pavement. Another alternative is to use saw cut beams from the pavement, but this may not always be possible owing to the difficulty of obtaining these samples and the resulting damage to pavement integrity.

Considerations for Future Research

Current pavement design methods for rigid pavements made with Portland Cement Concrete assume that loss of strength over time is due to the accumulation of damage from fatigue cracking. Materials to be used in the concrete are supposed to be screened for alkali-aggregate reactivity, susceptibility to freeze-thaw damage, and sulfate attack. By using State DOT approved material sources for CC6, it was assumed that all these requirements had been met. The FAA ATRD Branch's experience with the concrete mixes for MRS2 and MRS3 indicate that this cannot be taken for granted. Small amounts of alkali-aggregate reactivity can be enough to reduce concrete strength over time, even if there are no visible signs of distress on the concrete surface. Procedures for screening of mix design materials should be reviewed, as well as the accuracy of the accelerated tests that are commonly used for this screening.

The occurrence of delayed ettringite formation (DEF) was also something that the ATRD Branch had not previously encountered. Further investigation should be made into this subject. A brief literature research indicates that DEF is a topic of interest in the USA and other countries, particularly on older concrete structures (see References 5 and 6 below). The FAA is currently developing design methods for extended pavement life beyond the traditional 20 year life-cycle. The durability of materials is an important consideration if pavements are expected to remain in service for 40 years or longer. The FAA is advised to review this materials distress, and perhaps conduct its own research of DEF.

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